Generation of Non-Stationary Ground Motions For Probabilistic Seismic Risk Analysis of Nuclear Power Plants

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SUMMARY:
Probabilistic seismic risk analysis is a popular method to evaluate the failure risk of nuclear power plants due to earthquake. In this method, fragility curves which is obtained by performing non-linear time history analysis with artificially generated strong motions, are used to express the conditional probability of failure of a structure or component under a given ground motion parameter such as peak ground acceleration (PGA) or spectral acceleration. However, in most non-linear time history analyses, the non-stationarity in time and frequency of ground motions are often ignored. To address above problem, a method to generate ground motions with non-stationarity both in time and frequency is proposed for probabilistic seismic risk analysis of based isolated nuclear power plants. It is shown that the proposed method is more efficient and reasonable which helps to ensure a more reliable evaluation of failure risk for nuclear power plants.

Keywords: nuclear power plant, probabilistic seismic risk analysis, earthquake ground motion, non-stationarity

1. INTRODUCTION

Seismic safety of nuclear power plants (NPPs) is of great social and economic importance. Performance of NPPs in recent catastrophic earthquakes raises the reconsideration of some issues in seismic risk assessment. During the past 30 years, probabilistic seismic risk analysis has been applied as a popular method to evaluate the failure risk of NPPs due to earthquake (Huang 2010). In this method, fragility curves are used to express the conditional probability of failure of a structure or component under a given ground motion parameter such as peak ground acceleration (PGA), spectral acceleration (SA), peak ground velocity (PGV) or Arias intensity. Currently, there are two types of approaches available to obtain fragility curves. One is empirical, the other is analytical. The empirical approach is based on observed damage data collected from past earthquakes and regression analysis or maximum likelihood estimation is often used (Shinozuka 2000). In the empirical approach, the variation of structural parameters and ground motions is not considered and it may be inapplicable in seismic regions which do not have enough occurrences of earthquakes. However, the analytical approach, which is based on numerical simulation or stochastic analysis on structures subject to artificial records or stochastic models of earthquakes, is able to consider the variation of structural parameters and ground motions and can be efficiently applied to seismic risk assessment of structures without sufficient seismic experience. Therefore, the analytical approach is more widely used in recent years and usually the Monte Carlo simulation and the incremental dynamic analysis (IDA) are used in this approach.

The analytical approach commonly consists of four steps: (1) select a set of actual and/or synthetic ground acceleration records representing the seismicity at a given site, (2) scale each record by its PGA or SA to different excitation levels, (3) calculate maximum system response to scaled records, and (4) estimate the system fragility at each excitation level by the ratio of the number of times the maximum response exceeds a critical level to the total number of records. From above steps, it is clear that to consider the reasonability as much as possible when selecting actual and synthetic ground acceleration records for non-linear time history analysis is a key point to obtain convincing fragility curves.

As for the reasonability of ground motions, parameters regarding the conventional properties in amplitude, frequency and duration are often considered as controlling factors to evaluate the reasonability. However, it has been shown in many researches that the nonstationary properties in amplitude (intensity) and frequency content of ground motions also have significant influence on nonlinear response of structures. There are currently several notions available to describe such nonstationary properties, i.e. envelope is used for nonstationarity in amplitude, zero-crossing rate and instantaneous frequency are used for nonstationarity in frequency content (Li 1999, Dong 2010) and time varying spectrum is used for nonstationarity both in amplitude and frequency content (Dong 2010). Unfortunately, so far their practical applications in selecting and synthesizing ground motions for seismic risk assessment of structures are rarely reported.

To investigate the effect of nonstationarity of ground motions on seismic risk assessment of structures, a method for generating ground motions with nonstationarity both in amplitude and frequency content is proposed in this paper. Firstly, the envelope model and the instantaneous frequency model are used to describe the nonstationarity of ground motions. Then, the attenuation laws of model parameters and the regressive relations between model parameters and PGA are adopted to generate ground motions. Finally, the generated ground motions are applied to seismic risk assessment of nuclear power plants. The analysis results of ground motions with and without nonstationarity are compared and discussed.

2. MODELLING OF NONSTATIONARITY OF GROUND MOTIONS

In this paper, envelope and instantaneous frequency are respectively used to describe the nonstationarity in amplitude and frequency content.

The most commonly used envelope model is a piecewise function which is given by (Fig. 2.1)

$$E(t) = \begin{cases} t^2 / T_1^2 & 0 < t < T_1 \\ 1 & t_1 < t < T_1 + T_2 \\ e^{-c(t-T_1-T_2)} & t > T_1 + T_2 \end{cases} \tag{2.1}$$

where $E(t)$ is the envelope function, T_1 and T_2 are respectively the duration of rise and strong motion and c is the decay rate. Using the ground motions recorded worldwide since 1930, Dong (2010) proposed the attenuation relations of above envelope parameters given below

$$\log Y = c_1 M + c_2 \log(R+10) + c_3 R + c_4 T_g + c_5 + \varepsilon \tag{2.2}$$

where Y indicates the envelope parameters, M is the magnitude, R is the epicentre distance, T_g is the characteristic period (turning point) of response spectrum which is used to take into account the effects of R and local site condition and can be obtained using the method described in Dong (2010), ε is the residual with the standard deviation as σ_ε, c_1, c_2, c_3, c_4 and c_5 are regression coefficients as listed in Table 2.1.

The instantaneous frequency (IF) of ground motion, which is calculated according to its Hilbert transform, can be used to describe the nonstationarity of frequency content. The commonly used IF model is an exponential decay function given by

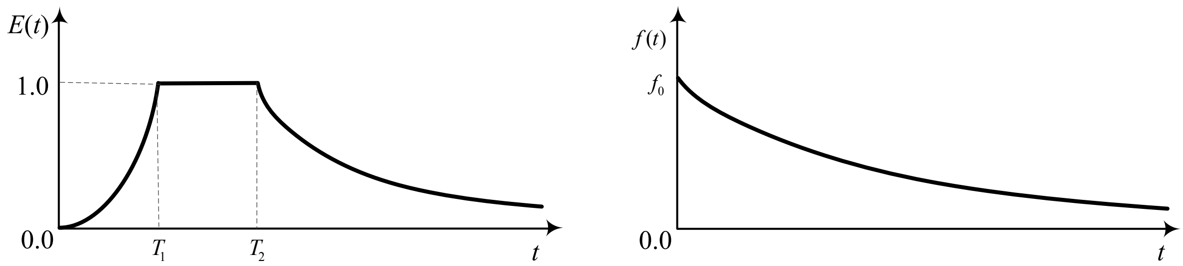
$$f(t) = f_0 e^{-\gamma t} \tag{2.3}$$

where $f(t)$ is the IF function, f_0 is initial frequency and γ is the decay coefficient. Using the same prediction relation as presented in Eqn. 2.2, Dong (2010) studies the attenuation rules of IF parameters and the corresponding regression coefficients are listed in Table 2.1.

It is to be noted that Li (1999) studied the nonstationarity of frequency content by using zero-crossing rate. The model of zero-crossing rate used is given by $\eta(t) = \eta_0 e^{-\nu t}$, where η_0 is initial zero-crossing rate and ν is the decay coefficient. It is clear that IF is more physically meaningful than zero-crossing rate which can directly describe the change of frequency content with time. There is an approximate equivalence between IF parameters and zero-crossing rate parameters

$$\eta_0 \doteq 2 f_0 \tag{2.4}$$

$$\nu \doteq \gamma \tag{2.5}$$



(a) Envelope model (b) IF model

Figure 2.1. Envelope model and IF model of ground motions

Table 2.1. Regression Coefficients of Envelope Parameters and IF parameters

Component	Y	c_1	c_2	c_3	c_4	c_5	σ_ε
Envelope	T_1	0.1767	1.0161	-0.0007	0.0695	-2.3635	0.4772
	T_2	0.2403	0.0150	0.0010	0.4871	-1.8619	0.4913
	c	-0.1107	-0.7053	0.0024	-0.1911	1.1010	0.3553
Instantaneous frequency	f_0	-0.0629	0.1223	0.0001	-0.4057	1.0551	0.1679
	γ	0.0283	0.1993	-0.0017	0.4615	-2.5858	0.5923

3. SYNTHESIS OF NONSTATIONARY GROUND MOTIONS

From the point of view of random process, the random model for ground motions in this paper is designated as

$$a(t) = A(t) \cdot F(t) \cdot x(t) \tag{3.1}$$

where $a(t)$ is the ground acceleration, $A(t)$ means amplitude modulation using the envelope model in Eqn. 2.1, $F(t)$ means frequency modulation using the IF model in Eqn. 2.3, $x(t)$ is the realization of a stationary random process. Here $x(t)$ is defined as the summation of trigonometric series with statistically independent phase angles

$$x(t) = \sum_{k=1}^{n} C_k \cdot \cos\left(\omega_k t + \phi_k\right) \tag{3.2}$$

where n is the total number of frequency bins, the variable k indicates the kth frequency bin $\omega_k = k \cdot \Delta\omega$ with $\Delta\omega$ as the frequency interval, $\Delta\omega_k = 1/T$ and T is total duration of $x(t)$, C_k is the amplitude for ω_k, ϕ_k is independent random phase uniformly distributed in the range of (0, 2π). Based on above model and given envelope and IF parameters, the method to synthesize nonstationary ground motions compatible with a specific (target) acceleration response spectrum consists of following steps:

1. Transform the target spectrum $S_a^T(\omega_k)$ obtain corresponding the power spectrum $S(\omega_k)$ by using

$$S(\omega_k)=\frac{\zeta}{\pi\omega_k}\cdot\left[S_a^T(\omega_k)\right]^2\cdot\frac{1}{\ln\left[\dfrac{-\pi}{\omega_k T}\ln(1-P)\right]} \tag{3.3}$$

where ζ is the damping ratio, P is the probability of exceedance varying between 0.1 and 0.15 usually.

2. Obtain the initial amplitude C_k by using $C_k=\sqrt{4\cdot S(\omega_k)\cdot\Delta\omega}$.

3. Generate the random phase angle ϕ_k which is uniformly distributed in (0, 2π).

4. Synthesize the stationary random process $x(t)$ using Eqn. 3.2.

5. Modulate $x(t)$ in amplitude and frequency as presented in Eqn. 3.1 to generate the nonstationary ground motion $a(t)$. Here, the envelope model is used for amplitude modulation. Using the equivalence between the parameters of IF and zero-crossing rate as presented in Eqns. 2.4 and 2.5, the method based on zero-crossing rate proposed by Li (2000) is used for frequency modulation.

6. Calculate the response spectrum of $a(t)$ and compare it with the target spectrum by calculating the average relative error e between them

$$e=\sum_{k=1}^{n}\left|\frac{S_a(\omega_k)-S_a^T(\omega_k)}{S_a^T(\omega_k)}\right|\Big/ n \tag{3.4}$$

where $S_a(\omega_k)$ is the response spectrum of $a(t)$.

7. Repeat steps 2 to 6 iteratively until e is below a threshold (e.g., 10%). In each iteration, the amplitude C_k is adjusted by

$$C_k^{i+1}(\omega_k)=\frac{S_a^T(\omega_k)}{S_a(\omega_k)}\cdot C_k^{i}(\omega_k) \tag{3.5}$$

where the superscript i and i+1 respectively indicate the ith and (i+1)th iteration.

Following steps mentioned above, a set of nonstationary ground motions, which are compatible with the target response spectrum, can be synthesized by changing the seeds of random number when generate the random phase. If the frequency modulation in step 5 is ignored, above steps summarize the implementation of the common method for generating ground motions based on trigonometric series without considering the nonstationarity of frequency content.

4. PROBABILISTIC SEISMIC RISK ANALYSIS OF A BASE ISAOLATED NPP

In this section, ground motions generated using above method are applied to seismic risk assessment of a base isolated nuclear power plant. The analysis results of ground motions with/without nonstationarity are compared and discussed.

4.1. The NPP Model

The section view of the base isolated NPP and its corresponding stick model for dynamic analysis are presented in Fig. 4.1 (Ali 2012). The model is 65.84 m in height and consists of 15 nodes and 14 elements. The average translational mass for each node is about 220×10^3 kg and the mean area and moment of inertia of each element respectively 168.69 m^2 and 3.98×10^4 m^4. A zero-length elastomeric bearing element with bilinear hysteresis is used to simulate the base isolator (Fig. 4.2), the initial stiffness K_u is 6.1×10^5 kN/m, the yield force F_y is 2.7×10^3 kN and the post yield stiffness K_d=0.167K_u. The natural period of the model is 1.35 s and the response spectrum with PGA of 0.2 g specified in U.S. Nuclear Regulatory Commission (NRC) Regulatory Guide 1.60 is adopted (NRC 1973). According to Naeim (1999), the allowable displacement of the isolator is 0.16 m. When generating ground motions, the epicentre distance of 25 km is considered. The software framework OpenSees is used to implement nonlinear time history analysis. For more details about the NPP model, readers can refer to Ali (2012) and Lee (1999).

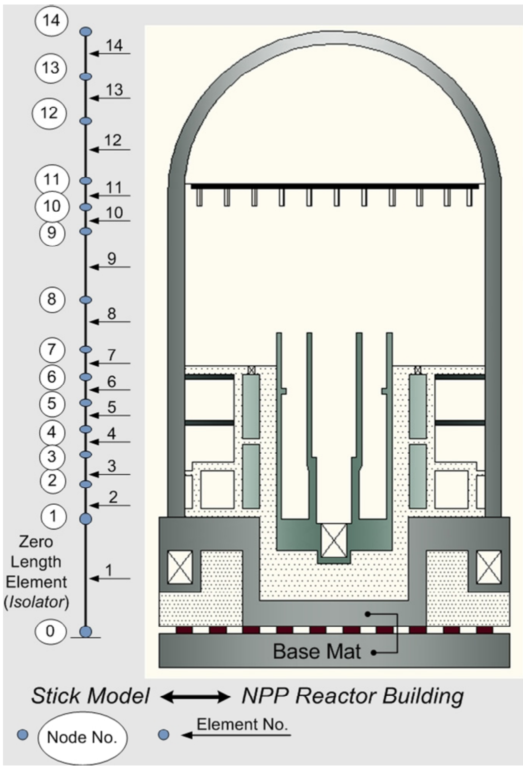


Figure 4.1. The section view of the base isolated NPP and its corresponding stick model

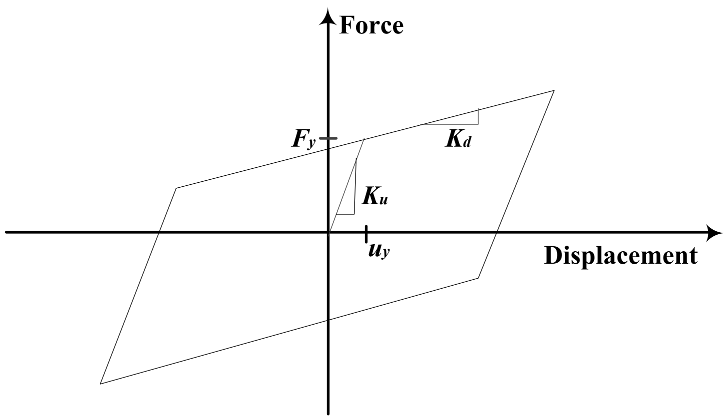


Figure 4.2. Bilinear hysteretic model of the base isolator

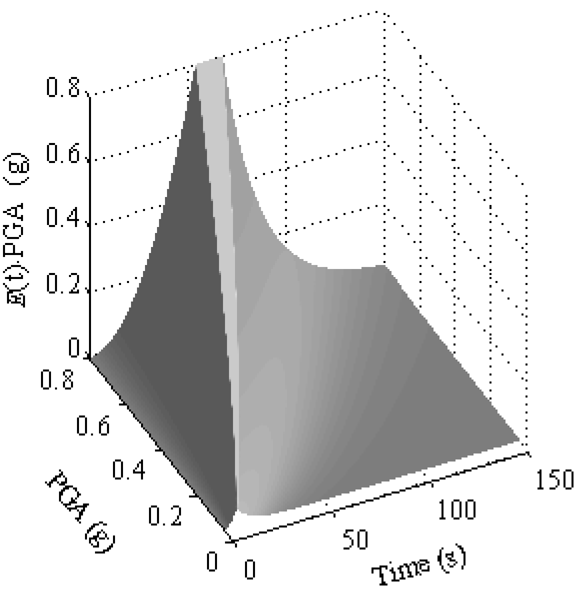
4.2. Generation of Ground Motions

To obtain fragility curves, the PGA of ground motions are scaled from 0.025 to 0.8 g with a constant increment of 0.025 g. Two types of ground motions, namely Type I and Type II, are defined. For Type I ground motions, only one set of 100 ground motions are generate using the envelope parameters determined according to the PGA of 0.2 g, the change of envelope parameters with PGA as well as the nonstationarity in frequency is not considered. For Type II ground motions, corresponding to each PGA bin a set of 100 ground motions are generated with considering the change of envelope and IF parameters with PGA. When generating ground motions the prediction equation of PGA (in cm/s^2) $\ln \text{PGA} = 0.4 + 1.2M - 0.76 \ln \sqrt{R^2 + 10^2} - 0.0094\sqrt{R^2 + 10^2}$ by Baag (1998) is used to build the relationship between envelope and IF parameters and PGA. The envelope and IF curves used are plotted in Fig. 4.3 which depicts their changing trends with PGA.

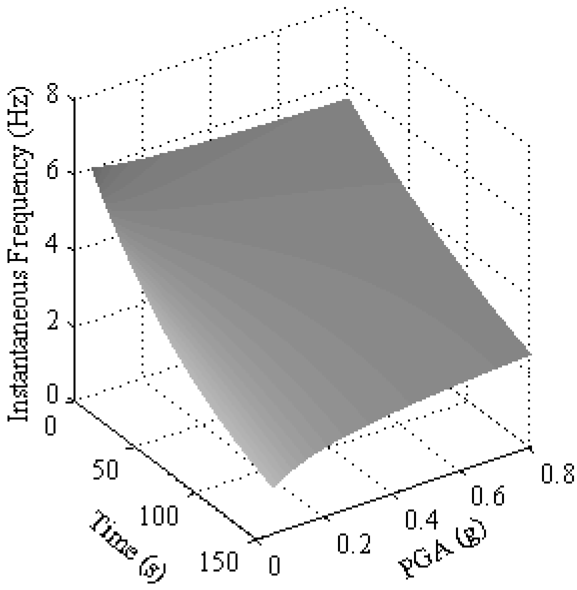
Fig. 4.4 shows the acceleration time history and time varying spectrum of a Type I ground motion. The sample mean of the normalized spectral acceleration is also demonstrated to evaluate the compatibility with the spectrum specified in NRC Regulatory Guide 1.60. Fig. 4.5 shows the acceleration time history, time varying spectrum and sample mean of the normalized spectral acceleration for Type II ground motions with PGA of 0.2, 0.4 and 0.8 g. It is clear that the response spectra of both types of ground motions are well compatible with the spectrum specified in NRC Regulatory Guide 1.60. Since the envelope is constant with PGA and the nonstationarity in frequency is not considered, the energy distribution in frequency domain of Type I ground motions keeps unchanged with time and PGA. The energy-frequency distribution of Type II ground motions changes obviously with time and PGA, since both change of envelope and IF with PGA is considered when generating ground motions.

4.2. Seismic Risk Analysis of NPP

Nonlinear time history analysis is repeated for each ground motion and PGA bin. The displacement of base isolator is used to evaluate the performance of the NPP. Three limit states, termed as State 1, 2 and 3, corresponding to the displacement of base isolator of 0.5, 1.0 and 1.5 times of the allowable displacement, are considered to obtain the fragility curves of the NPP. The fragility curves for above



(a) Envelope

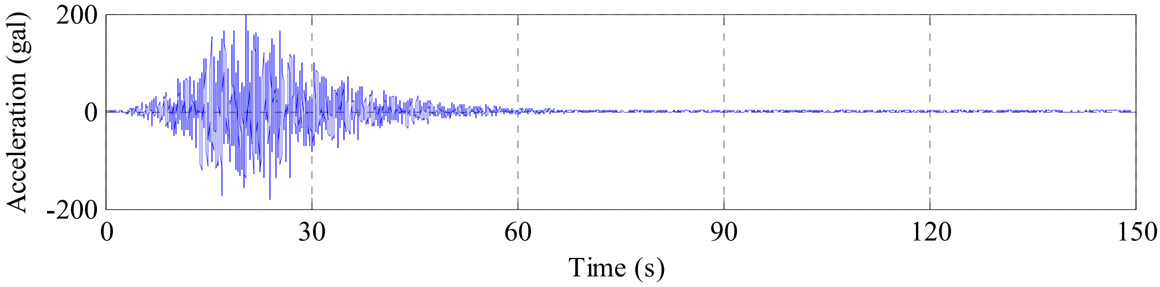


(b) Instantaneous frequency

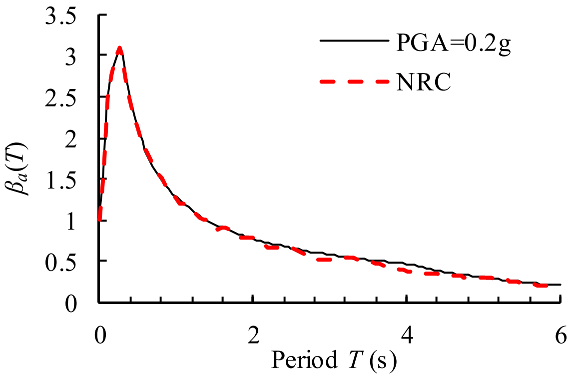
Figure 4.3. Change of envelope and instantaneous frequency with PGA

states under two types of ground motions are given in Fig. 4.6. It is clear that the probability of all the states for Type II ground motions is larger than that for Type I ground motions which indicates the higher seismic risk of the NPP under Type II ground motions.

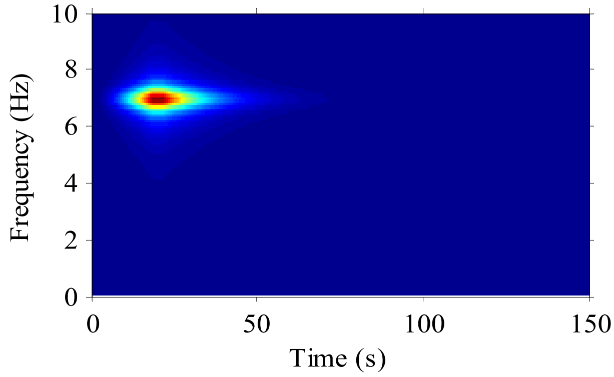
Using the seismicity parameters of source model C in Nakajima (2007), the seismic hazard curve of the site is obtained as given in Fig. 4.7. By combining the seismic hazard curve with fragility curves, the probability distribution of each limit state is shown in Fig. 4.8. The peak values in probability distribution of all the limit states under Type II ground motions are larger than those under Type I ground motions. The PGA corresponding to the peak value of the probability distribution curve is called dominant PGA. For all limit states, the dominant PGAs of Type II ground motions are smaller



(a) Time history of ground acceleration

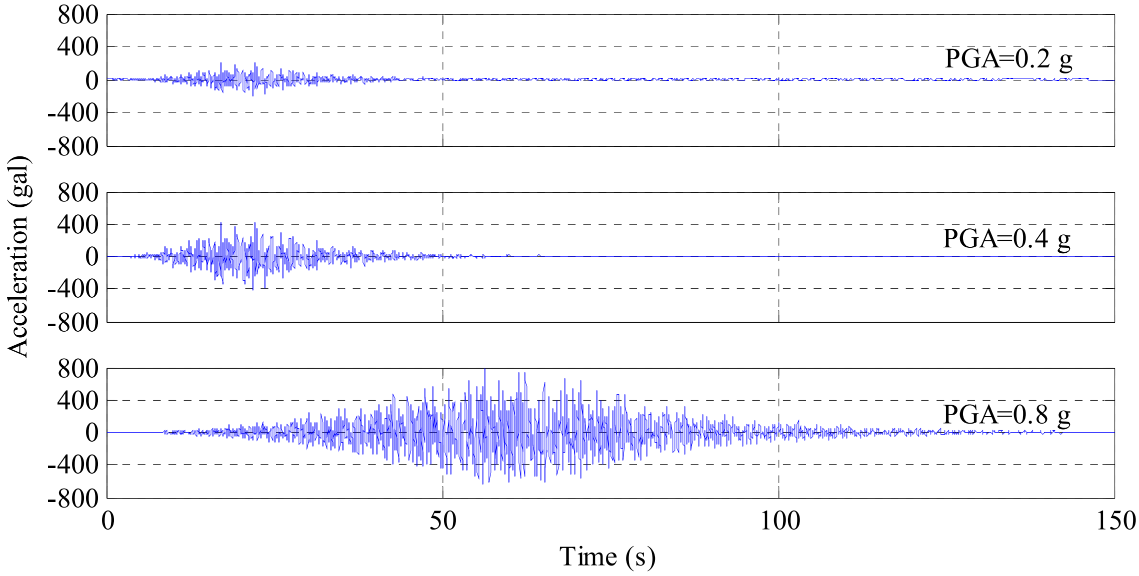


(b) Normalized spectral acceleration

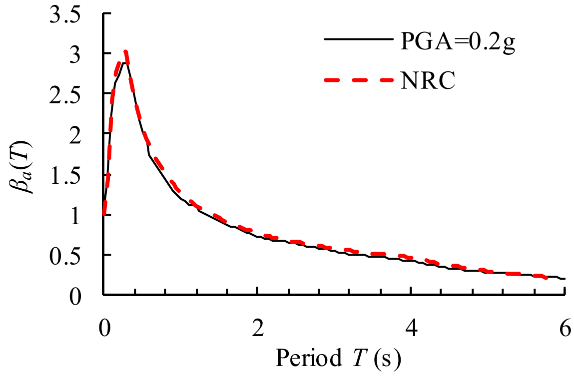


(c) Time varying spectrum

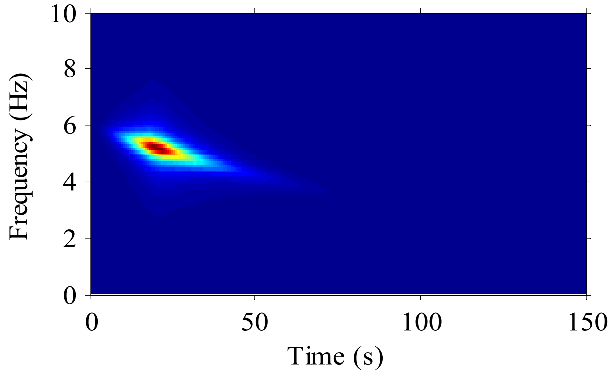
Figure 4.4. Time history, spectral acceleration and time varying spectrum of Type I ground motion



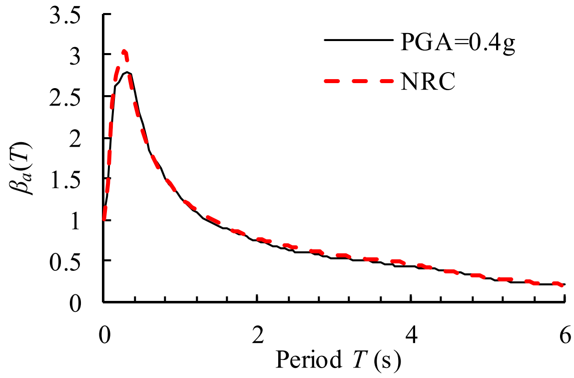
(a) Time histories of ground accelerations



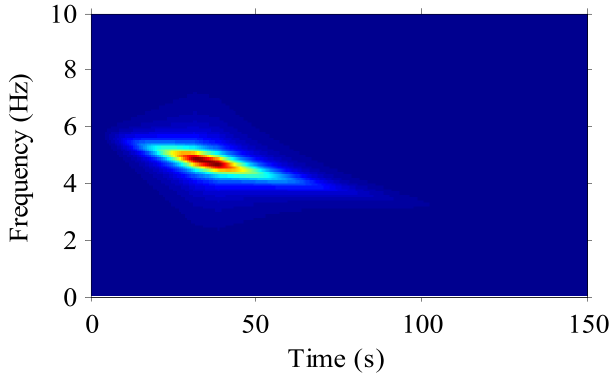
(b) Normalized spectral acceleration (PGA=0.2g)



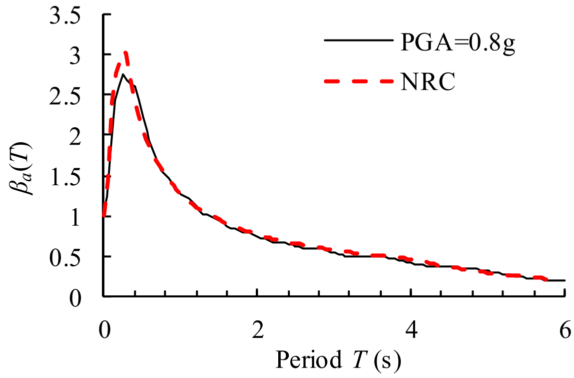
(c) Time varying spectrum (PGA=0.2g)



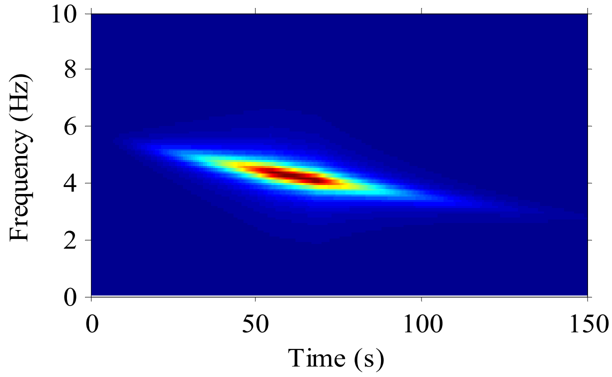
(d) Normalized spectral acceleration (PGA=0.4g)



(e) Time varying spectrum (PGA=0.4g)



(f) Normalized spectral acceleration (PGA=0.8g)



(g) Time varying spectrum (PGA=0.8g)

Figure 4.5. Time history, spectral acceleration and time varying spectrum of Type II ground motions

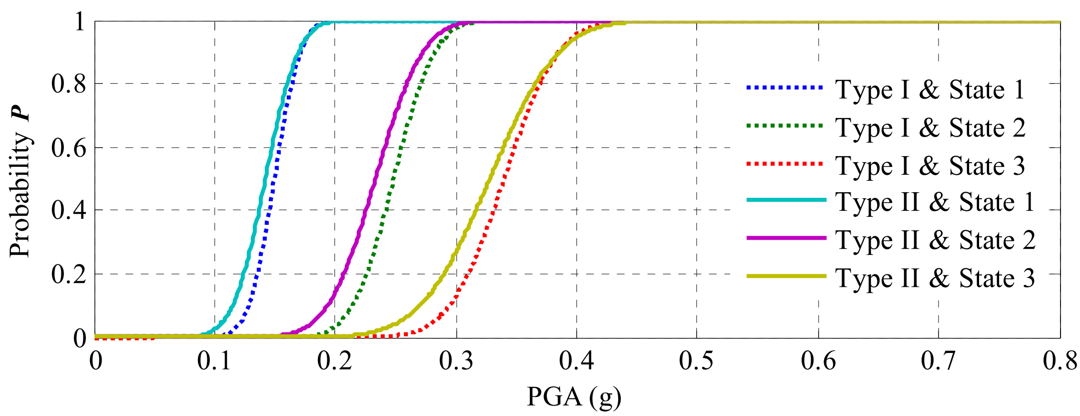


Figure 4.6. Fragility curves of the NPP under two types of ground motions

than their counterparts of Type II ground motions. Using the method described in Huang (2010), the annual probability of each limit state is calculated and corresponding results are also in Fig. 4.8. It is clear that the annual probability of all the limit states under Type II ground motions is considerably larger than that under Type I ground motions, which implies using ground motions without considering the change of envelope and IF with PGA for fragility analysis may lead to the underestimation of seismic risk of the NPP.

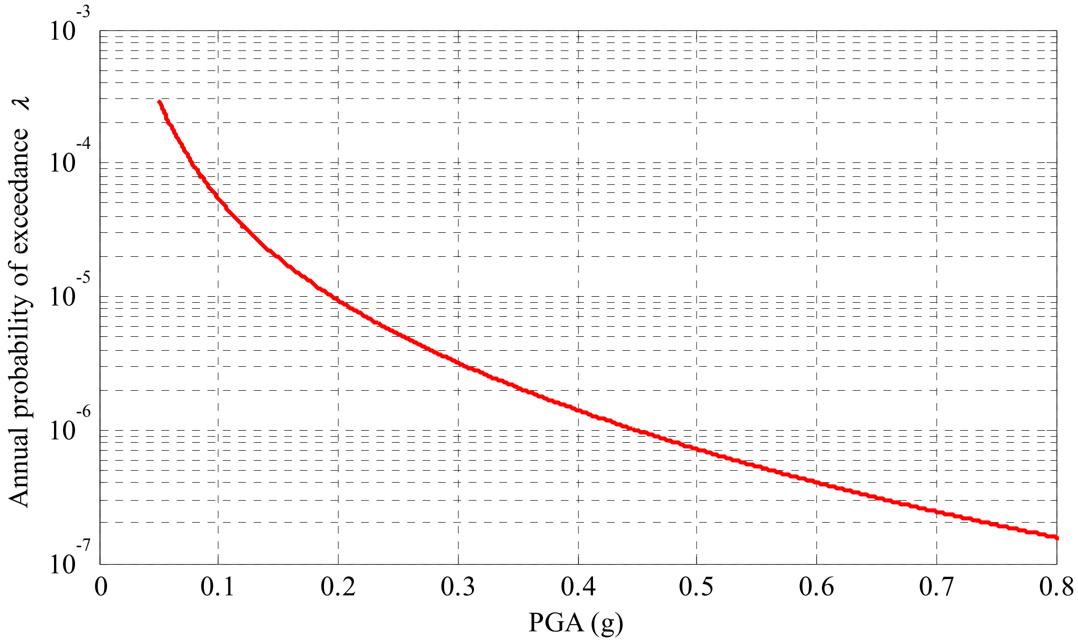
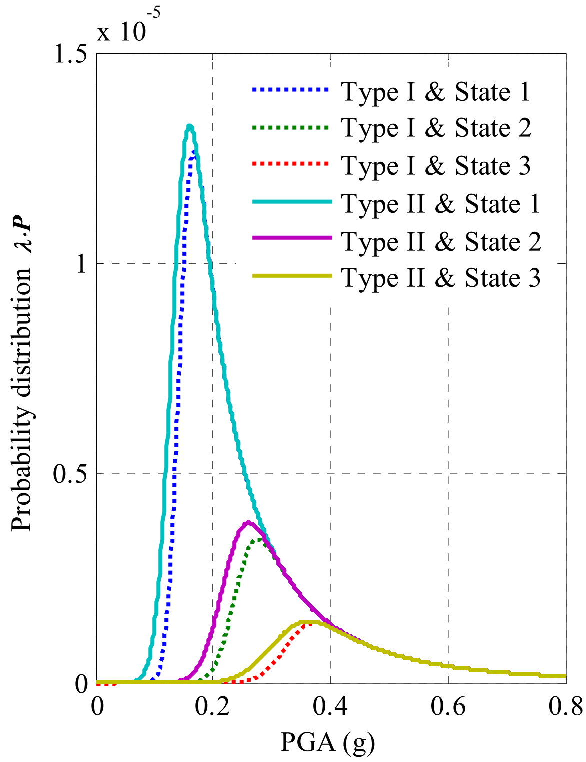
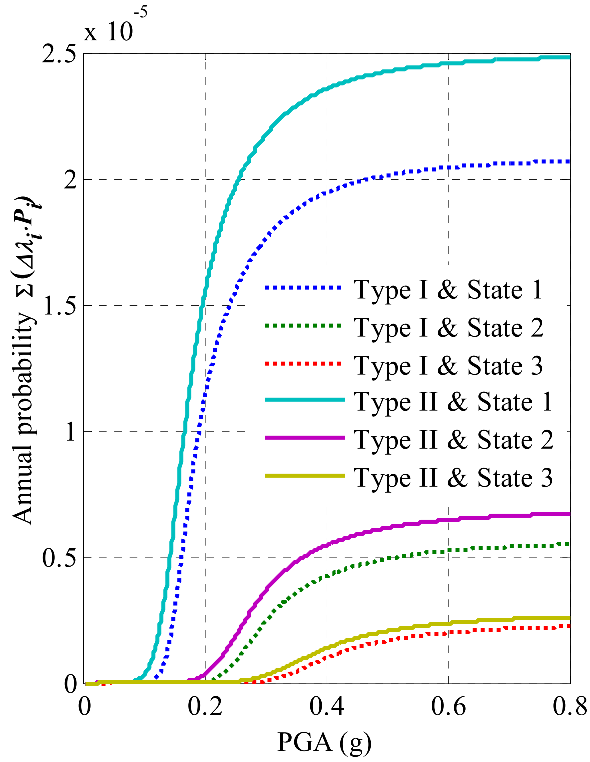


Figure 4.7. Seismic hazard curve



(a) Probability distribution of limit states



(b) Annual probability of limit states

Figure 4.8. Probability distribution and annual probability of performance states

4. CONCLUSIONS

The nonstationary properties in amplitude (intensity) and frequency content of ground motions also have significant influence on nonlinear response of structures. To investigate the effect of nonstationarity of ground motions on seismic risk assessment of structures, a method to generate ground motions with nonstationarity both in time and frequency is proposed for probabilistic seismic risk analysis of a based isolated nuclear power plant. The analysis results of ground motions with and without nonstationarity are compared and discussed. It is found that using ground motions without considering the nonstationarity for fragility analysis may lead to the underestimation of seismic risk of structures. It is necessary to consider the nonstationarity when selecting and synthesizing ground motions for seismic risk analysis of structures. Besides the nonstationarity of ground motions, lessons learned from recent earthquakes have also shown the effects of velocity pulse, long period property and spatial variation on structural performance, which will be the directions for future research.

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